DAVISON RANCH FLOODPLAIN DELINEATION
PRAIRIE CREEK, HUMBOLDT COUNTY, CALIFORNIA

REDWOOD
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PARKS

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REDWOOD NATIONAL AND STATE PARKS

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INTRODUCTION

This hydrologic/hydraulic analysis of flooding characteristics was done to assist with Redwood National and State Parks planning activities in and around the Davison Ranch and B-mill deck site along Prairie Creek in Humboldt County, California (Fig. 1). As described in the Davison Ranch Development Concept Plan (DCP) and Environmental Assessment (EA) (RNSP, 1996), five development alternatives are being considered for the area. The alternatives range in scope from no action, which leaves the former ranch in its present condition including existing buildings and uses, to development of fairly substantial visitor facilities. At present, the preferred alternative is to construct hiking, biking, and equestrian trails which would tie into existing and planned visitor facilities on adjacent or nearby areas.

While development of trails is considered to be an "excepted action" which would not require floodplain delineation or development of a floodplain management plan (NPS, 1993), Redwood National and State Parks (RNSP) plan to retain for use the buildings (barns, residence, etc.) from the former ranch as well as some of the facilities from the former mill site. It is this aspect of the park's management of the Davison Ranch which creates the need for this floodplain delineation. While this is not meant to be an exhaustive, highly precise analysis, it will serve as a guide to assist with minimizing potentially hazardous conditions associated with flooding. Should future plans include more intensive use or development of the Davison Ranch, a more precise floodplain analysis may be required.

As also mentioned in the DCP, RNSP is currently undertaking development of a general management plan (GMP). This plan will incorporate elements of the DCP and will similarly rely on this floodplain delineation and analysis in consideration of management alternatives. In conjunction with this floodplain delineation, the park is also undertaking a wetlands delineation for the Davison Ranch. This project will also include development of wetland restoration alternatives aimed at reversing the effects of prior drainage works associated with former ranch operations.

In 1994, a preliminary floodplain assessment indicated that much of the valley area in the vicinity of the Davison Ranch was likely in the regulatory floodplain, but certain key areas were questionable. Because of the presence of both existing structures and planned improvements in several of these questionable areas, this more quantitative approach was warranted. In preparation for this, as well as for other planning purposes, a detailed (4 foot contour interval) map was prepared by the Denver Service Center (DSC). This map was utilized for determining channel slopes through the study reach and for delineating the 100-year floodplain at the Davison Ranch.

Channel cross sections were surveyed and other site-specific quantitative information was collected to provide input data to a hydraulic model for estimating the water surface profile of the 100-year flood. Peak discharge for the 100-year flood was estimated from magnitude-frequency relations at nearby gaging stations. Using these data along with flood heights observed for a 10-year flood which occurred in December, 1995, a reasonably well-constrained assessment of flood hazards was possible.

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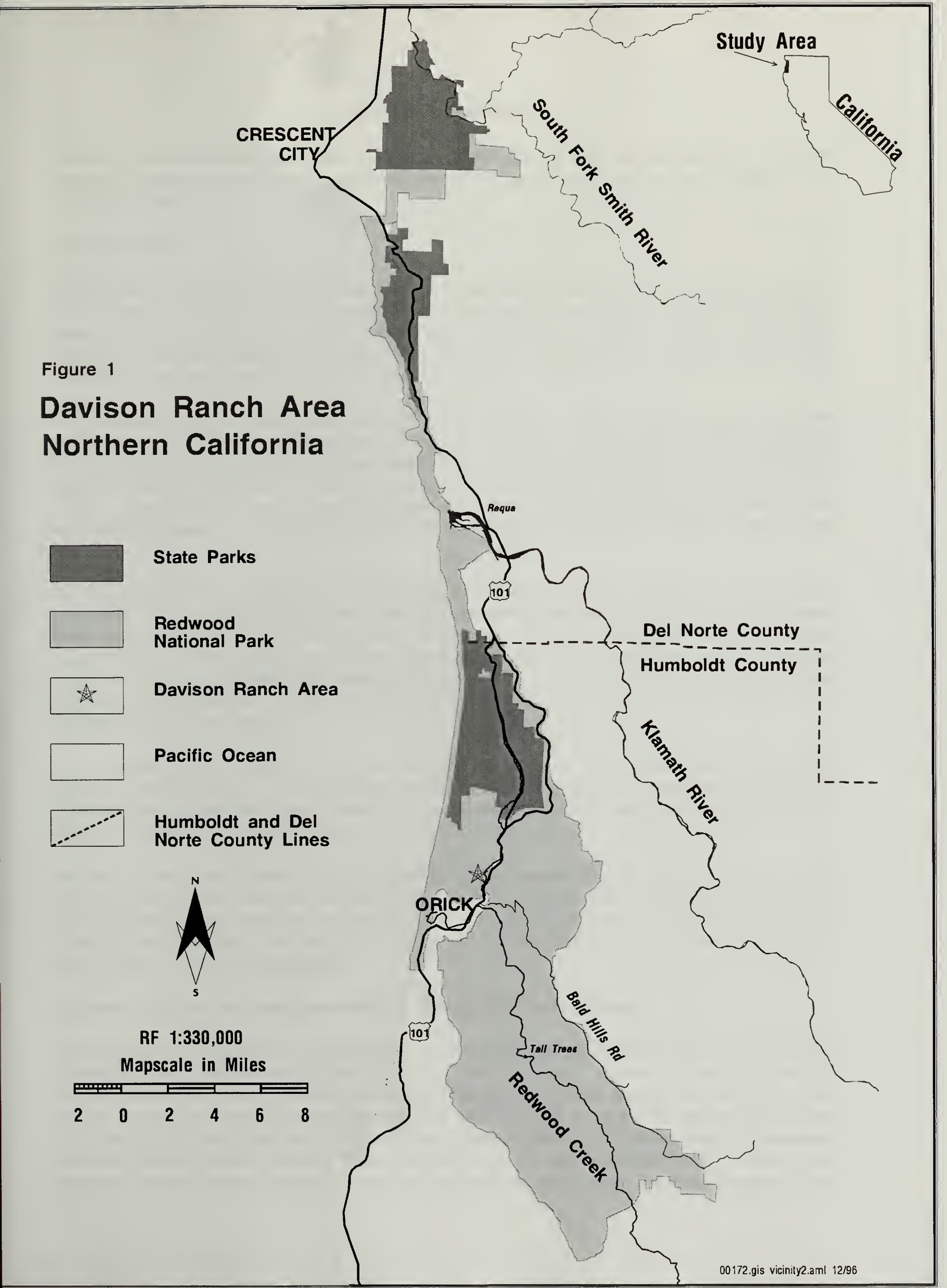


Figure 1

Davison Ranch Area
Northern California

SITE DESCRIPTION

Figure 2 shows the study area and several pertinent natural and cultural features. The area covered in this analysis includes approximately 104 acres of valley bottom land on either side of Prairie Creek.

Geomorphology

Prairie Creek through the Davison Ranch is a low gradient stream which meanders through the valley with a sinuosity of 1.2 (channel length/valley length) and has an overall slope (S) of about 0.0014 or 0.14% (5.3 feet of drop over 3857 feet of channel length). Two distinct segments of differing slope exist within the study reach: that above the Davison Road bridge (S=0.0010) and that below (S=0.0017) (see Fig. 1).

The drainage area contributing to the project site is 36.6 mi^2. Mean channel width is 94 feet, ranging from 176 feet in the upper part of the study reach to about 60 feet in the lower part. Channel depth averages about 8.5 feet, ranging from 4.9 feet in the upstream end of the study reach to nearly 13 feet in the downstream end. The valley varies in width from about 600 feet at the upstream end of the study area to over 2000 feet in the central part of the area, nearly all of which appears to be in the floodplain. The length of the valley area studied is about 4300 feet.

For the most part, the channel consists of a single thread with small gravel bars on the insides of bends. Two exceptions are located near the upstream and downstream ends of the study reach, where the channel is split between a main channel, which carries most of the flow, and a secondary channel, which only carries storm runoff. While the two wider sections resulted most likely from accelerated bank erosion at some point in the past, most of the channel now appears relatively stable. Evidence of recent bank erosion is limited to small areas at the outsides of the major bends.

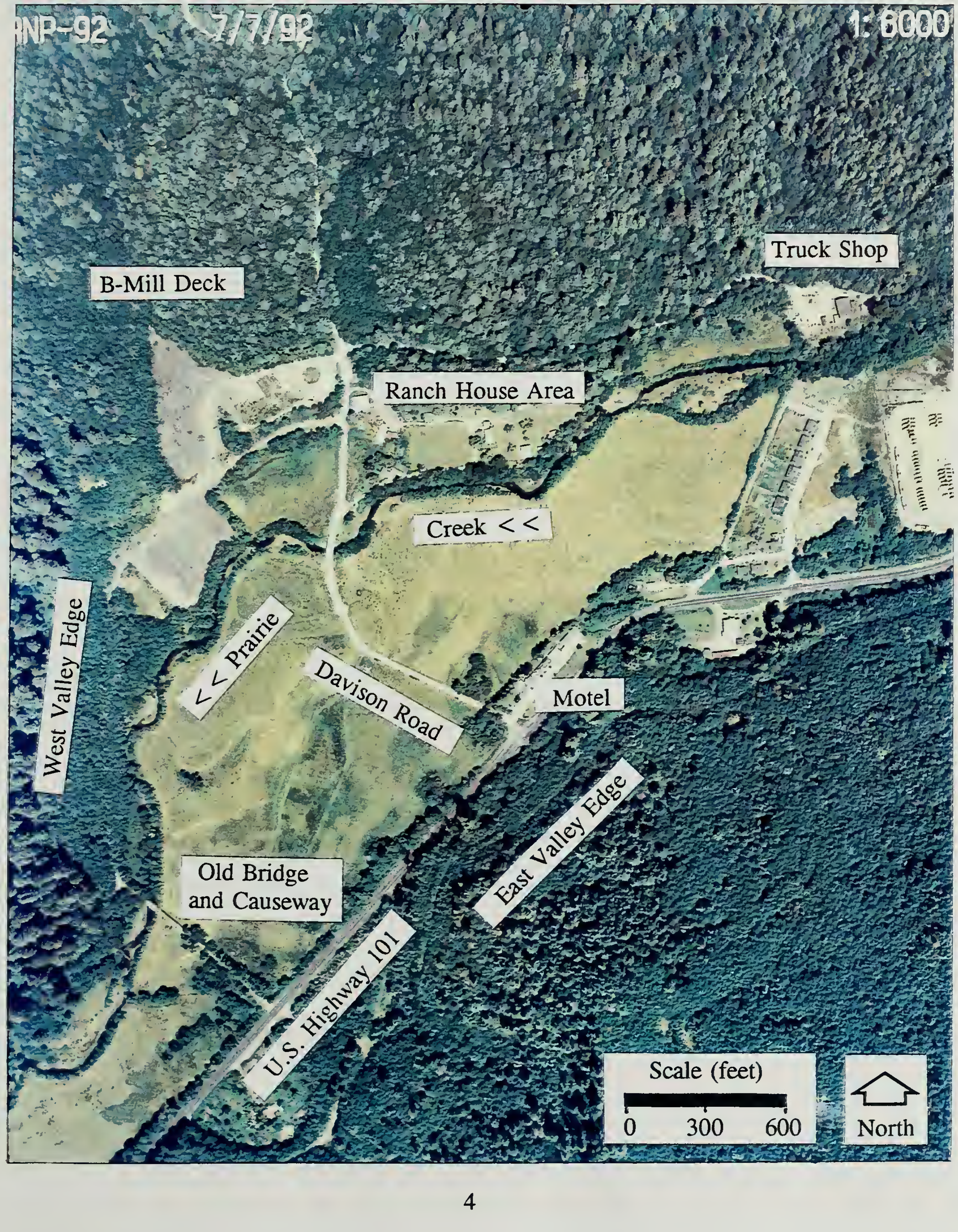
The lowlands on either side of Prairie Creek are generally flat with occasional topographical features consisting of: 1) old meander scars which have become seasonal or perennial marshes, and 2) natural levees adjacent to the present course of the creek which are up to 3 feet higher than floodplain surfaces farther away from the banks of the creek.

Because Prairie Creek tends to hug the west side of the valley, the floodplain on the east side of the creek is much wider than on the west side. While the original floodplain included lands along the eastern valley edge, Highway 101 has effectively cut this area off from the present Prairie Creek floodplain. The lands to the east of Highway 101, comprising about 10 acres in total, are not included in this analysis.

Pertinent Cultural Features Representing Floodplain Encroachments

All of the study area except the B-Mill Deck is characterized as relatively flat ground typical of stream-adjacent floodplain or terrace surfaces. The B-Mill deck appears to be an alluvial fan deposit emanating from a tributary drainage flowing into Prairie Creek from the west. The tributary stream is presently routed beneath the deck through a culvert for the lowest 270 feet of its length. The deck surface comprises the highest ground in the study area and is paved over with asphalt, a relict from its former use as a mill site and log transfer and loading deck. While the upper (western) margin of the deck is clearly an alluvial fan, the lower margins of the

Figure 2. Pertinent natural and cultural features at the Davison ranch, Redwood National and State Parks.



deck may have been elevated with fill material when the deck was constructed. This cannot be determined without soils analyses of the materials beneath the pavement. In either case, the bank height along the streamward edge of the deck (right bank) is several feet higher than on the opposite (left) bank.

The ranch house area (11 acres) is located on the floodplain to the west of Prairie Creek and extends west to the base of the hillslope and south to Davison Road, a county road which connects Highway 101 to Gold Bluffs Beach. The area is separated from Prairie Creek by a low (2-3 feet tall) levee constructed by the former residents. The levee ties into Davison Road, which effectively completes the levee system around the ranch house area.

Davison Road is owned and maintained by Humboldt County. The section through the Davison Ranch is a two-lane paved road which is elevated to varying degrees above the surrounding floodplain. The county bridge over Prairie Creek is a full-spanning structure (no piers) which was constructed fairly recently (within the last 15 years). The span of the bridge is 97 feet and the height of the bottom chord (bottom edge of the span) is about 13 feet above the channel bed. The road approaches were constructed from imported fill. They are both about 100 feet long, ramping up to the bridge deck which is about 18 feet above the channel bed. RNSP has constructed a split rail fence about 3 feet tall on both sides of Davison Road east of Prairie Creek.

Another, older bridge is located near the downstream end of the study area. This bridge was formerly used by the timber company to access the B-Mill deck, and is constructed of large redwood stringers and numerous redwood piers, both in the channel of Prairie Creek and on the floodplain to the east. The bridge span is 60 feet over the channel and another 120 feet over the floodplain. The bottom chord of the bridge is located about 13 feet above the channel bottom. The bottom chord of the bridge section spanning the floodplain to the east of the creek is suspended about 5 feet above the ground surface by 14 piers on roughly 8 foot centers.

A causeway (elevated roadway) connects the eastern end of the bridge to Highway 101. The causeway consists of an earthen fill, the surface of which is paved with asphalt and is about six feet higher than the elevation of the adjacent floodplain. It spans about 450 feet of the eastern valley floor to the point where it meets Highway 101. Highway 101 is located in the eastern margin of the Prairie Creek floodplain. The highway's road fill is elevated from about 6 to 8 feet above the floodplain and encroaches into the floodplain from 100 to 500 feet along the project reach. Where the causeway joins the highway, the encroachment from the highway alone is about 400 feet. Thus, the total encroachment onto the floodplain at the lower bridge is about 850 feet, or about 90% of the total natural floodplain width of 940 feet.

FLOOD HYDROLOGY

For this analysis, there were two peak discharges of interest: 1) the 100-year flood (or Q_{100})), and 2) the peak discharge of December 30, 1995.

Peak Discharge for the 100-Year Flood

Peak discharge for the Q_{100} was estimated from a flood frequency relation derived by the USGS for Little Lost Man Creek (USGS Gaging Station No. 11482468) based on 13 years of record (1975-1988, with 1983-84 missing). The estimate of the Q_{100}, performed in 1982 by the

USGS Water Resources Center (Sacramento, California) using a Log-Pearson Type III frequency distribution, was 1338 cubic feet per second (cfs). By dividing this flow by the contributing drainage area (3.46 mi^2), a flood peak of 387 cfs per square mile was calculated. This value was then multiplied by the contributing drainage area of Prairie Creek above the Davison Ranch (36.6 mi^2), yielding 14,164 cfs for the estimate of the Q_{100}. Generally, unit flood peaks decrease within increasing basin size. Because the USGS flood frequency estimates used here are transposed from a considerably smaller basin (about one- tenth the size of Prairie Creek at the Davison Ranch), the Q_{100} estimate is likely to be conservative (higher than actual). While the actual Q_{100} for the site may be closer to 300 cfs/mi^2, the higher value was used in favor of erring on the side of conservatism.

Flood of December 30, 1995

Peak Discharge: Based on data collected at gaging stations upstream on the main channel Prairie Creek (drainage area = 12.6 mi^2) and on Little Lost Man Creek (drainage area = 3.46 mi^2) (gaging stations presently maintained by RNSP), the flood of December 30, 1995 appeared to be about a 10-year recurrence interval flood (Log-Pearson Type III). The peaks recorded at these stations were both about 160 cfs per square mile. Multiplying this by the contributing drainage area gives a peak flow estimate of 5,860 cfs for Prairie Creek at the Davison Ranch for the December 30, 1995, flood.

Inundation: Following this flood, a field reconnaissance was done to observe flooding patterns and tie in high water marks (HWMs) to several cross sections used for this analysis. Field evidence (silt deposits, flood debris) indicated that virtually all of the valley bottom was flooded during this event. Figure 3 shows the areas inundated along with approximate water depths at several locations. Water depths, based on HWMs, ranged from a trace to about 4 feet over the lowest areas to the east of the natural levees along Prairie Creek.

The only large valley bottom area not flooded was the B-Mill deck. Other, smaller areas not flooded were the elevated fill approaches to the county bridge and the causeway connected to the old log bridge at the downstream end of the study site. Davison Road (excluding the east and west fill approaches) was covered by about 3 feet of water at peak discharge. On the segment of Davison Road east of the bridge, the combination of the slightly elevated roadbed, the bridge and raised fill approaches, and the split rail fence may have created a minor backwater effect. A large amount of woody debris was rafted up against the split rail fence on the upstream side of Davison Road, which effectively created a partial low dam across the left overbank area.

Significant depths and velocities over most of the inundated area were indicated by rafted logs and smaller woody debris stacked up against fences and vegetation and over the prairie surfaces upstream of Davison Road. An exception was the ranch house area, where inundation appeared to result from floodwaters backing up into the area across a low spot in the county road to the west of the bridge. Water depths ranged up to about 1 foot in the vicinity of the ranch house, except for an elevated piece of ground at the upstream end of this area (see Fig. 3). The artificial levee constructed by the former residents seems to have prevented direct flooding from the creek into the ranch house area. High water just barely overtopped the levee in several locations, as indicated by flow-swept grass and minor accumulations of flood debris. Consequently, the flood of December, 1995, provides a good upper limit on the flood protection derived by the levee. Any flows higher than about the Q_{10}

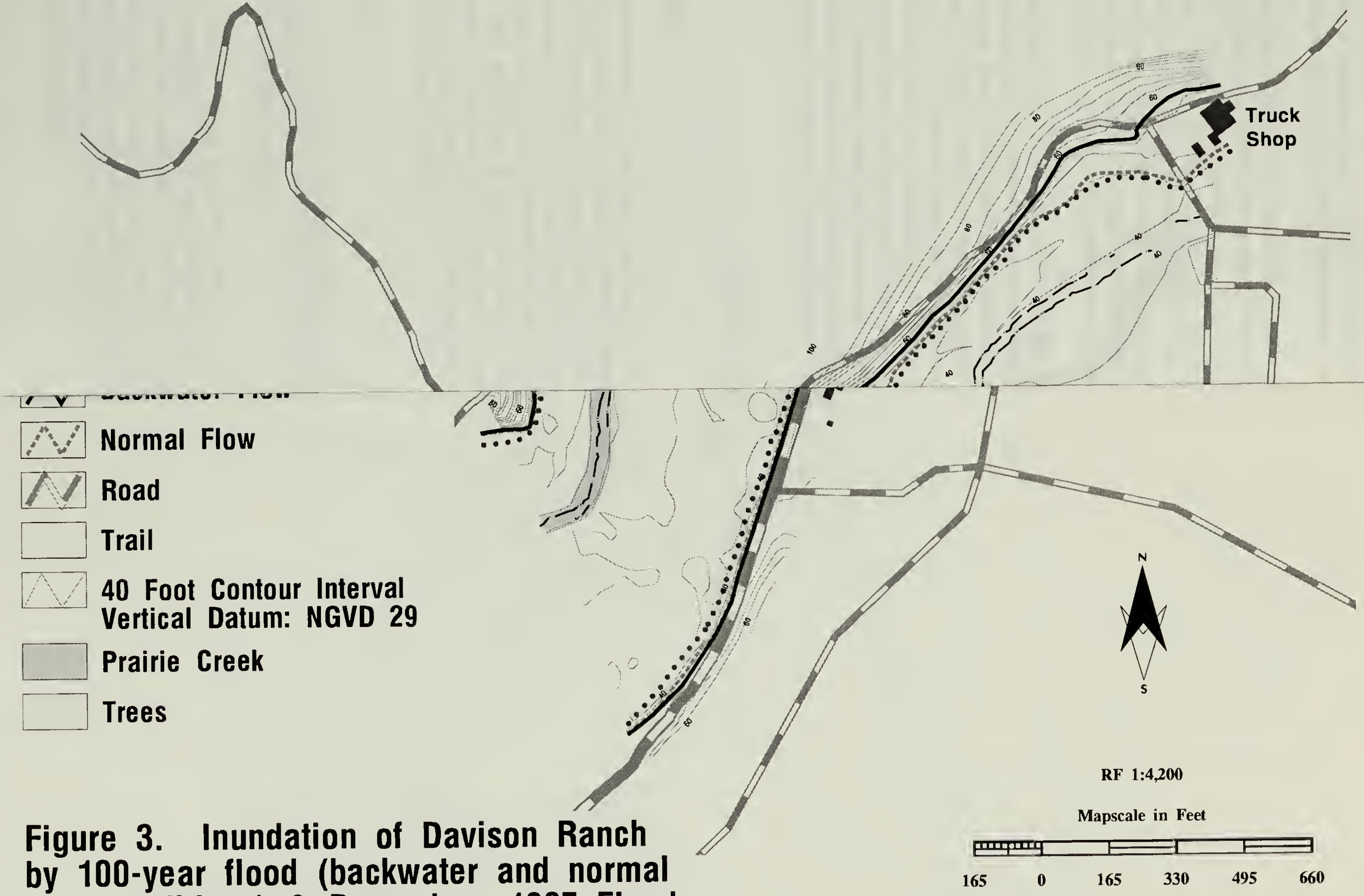


Figure 3. Inundation of Davison Ranch by 100-year flood (backwater and normal flow conditions) & December, 1995 Flood.

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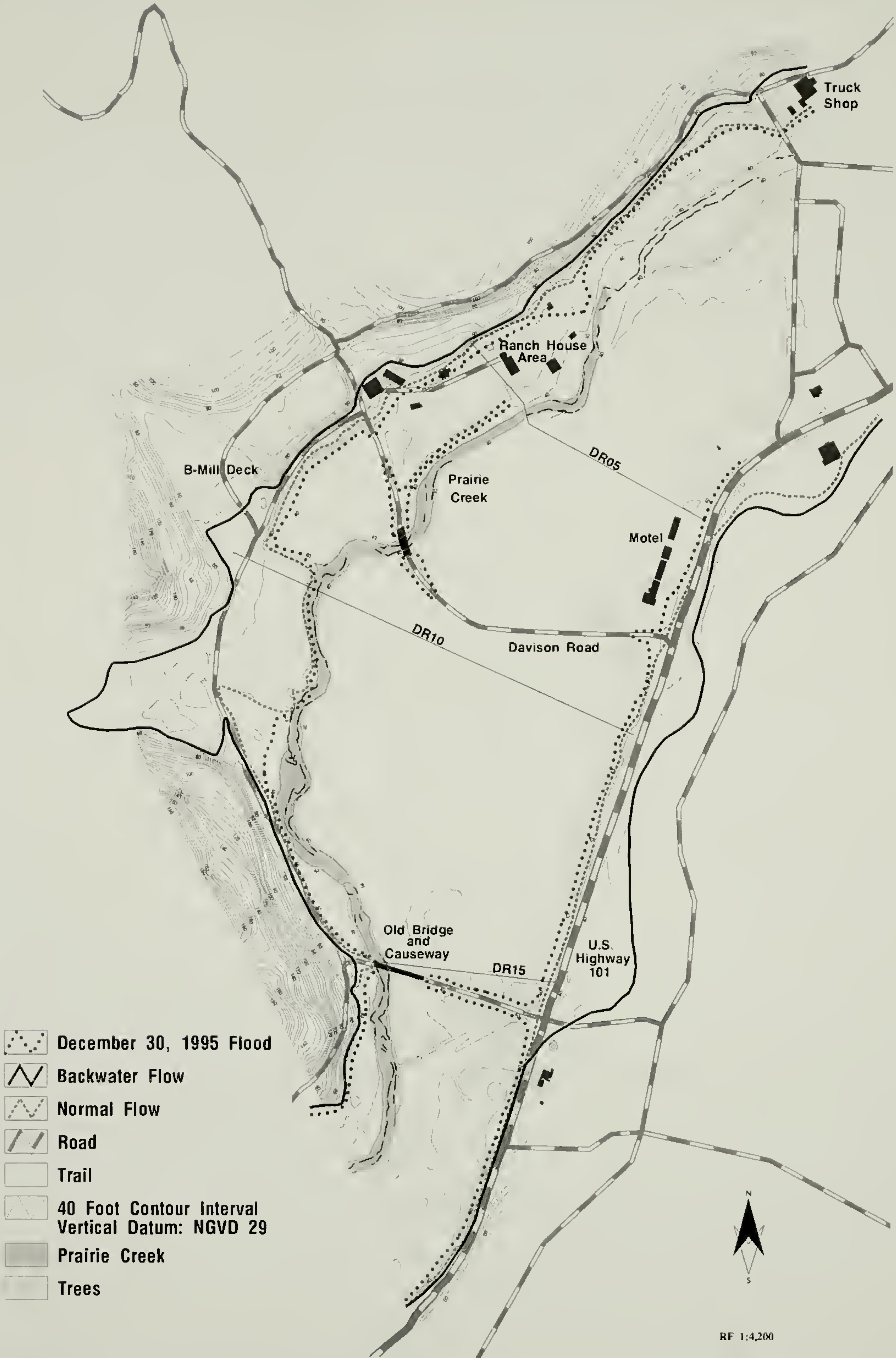


Figure 3. Inundation of Davison Ranch by 100-year flood (backwater and normal flow conditions) & December, 1995 Flood.

(6,000 cfs or greater) would result in widespread overtopping and perhaps erosion of the artificial levee, causing significant floodwater heights and velocities in the ranch house area.

Backwater Indications: The natural width of the valley decreases from the middle part toward the downstream end of the study area. The effective valley width has been further reduced by about 90%, as mentioned earlier, by both U.S. Highway 101 and the causeway to the east of the old log bridge, which truncate the eastern natural floodplain. This, combined with the roughness created by the bridge itself, comprises a major obstruction to flood flows and appears to have had a significant effect on flood water surface elevations at the downstream end of the study area. Comparison of HWMs just above and below the bridge indicates an abrupt drop of 1-1.5 feet in the high water surface of the December, 1995, flood.

In between the piers of the old bridge the ground surface is about 1.5 feet lower than adjacent floodplain surfaces upstream and downstream. This is most likely due to scour during floods. As moderate floods flows become severely constricted to pass beneath the bridge, flow conditions probably change from subcritical to critical. This may explain the apparent steepening of the water surface slope and much of the 1-1.5 foot drop in the observed water surface elevation at the old bridge. Unfortunately, no reliable HWMs could be surveyed within the 1000 foot reach upstream from the old bridge. Consequently, the conclusions regarding hydraulic conditions during the December, 1995, flood are somewhat speculative. However, as discussed later, significant backwater effects are a near certainty during larger floods because of the severe constriction of the floodway by US Highway 101 and the adjoining causeway.

HYDRAULIC MODELLING

Model Used

A one-dimensional model called the Manning equation was used for hydraulic modelling in this analysis. A commercial program called "Flowmaster", which uses the Manning equation, was used for computations and graphical output (Note: no endorsement is implied by the National Park Service by mention of any product names contained herein). While a step-backwater model, such as HEC-2, would be more appropriate to use for this floodplain delineation, it was felt that the additional effort and cost to develop the necessary data and run a more sophisticated model was not warranted. Should RNSP consider non-excepted actions in the future, re-analysis using HEC-2 or an equivalent model may be warranted. Model inputs and outputs are shown in Table 1.

Data Input

All input data, as well as model outputs, are shown in Table 1. **Discharge rates** used were for the Q_{10} (flood of December, 1995: 5,860 cfs) and Q_{100} (14,164 cfs). Derivation of these flood peaks was discussed earlier.

Three surveyed **cross sections** were used for modeling ground topography and flood water surface elevations. These were located near the ranch house (DR05), near the B-Mill deck (DR10), and just upstream of the old log bridge (DR15) (see Fig.3). The cross sections extended across the entire valley, including both the bankfull channel and the left (east) and right (west) valley overbank areas. Cross sections were surveyed in August, 1994, using a total station with elevation tie-ins to an air photo control survey conducted by Denver Service Center staff the previous year.

Table 1. Hydraulic computational inputs and outputs for modeling the Q_{10} (5,860 cfs) and Q_{100} (14,164 cfs) floods at three Prairie Creek cross sections in the Davison Ranch area, RNSP. Note: shaded cells contain calculated values; all other values were determined by other methods and used as inputs.

Case No.	Cross Section	Hydraulic Condition	Dis-charge (cfs)	Water Surface Slope	Rough-ness ("n")	Flow Area (sq ft)	Mean Velo-city (ft/sec)	WSEL (ft)
1	DR05	Observed (as is)	5,860 (Q_{10})	0.0010	0.085	3,813	1.54	45.40
[illegible]	DR05	Normal (road gone)	14,164 (Q_{100})	0.0010	0.062	6,003	2.36	47.45
[illegible]	DR05	Backwater (concave)	14,164 (Q_{100})	0.0017	0.178	11,389	1.24	52.37
[illegible]	DR15	Observed (as is)	5,860 (Q_{10})	0.0017	0.150	5,562	1.05	43.60
5	DR15	Normal (road gone)	14,164 (Q_{100})	0.0010	0.062	5,553	2.55	43.59
5	DR15	Backwater (concave)	14,164 (Q_{100})	0.0005	0.192	17,236	0.82	51.50
[illegible]	DR15	Observed (as is)	5,860 (Q_{10})	0.0017	0.029	1,043	5.62	41.10
8	DR15	Normal (road gone)	14,164 (Q_{100})	0.0017	0.062	4,270	3.32	42.29
9	DR15	Backwater (concave)	14,164 (Q_{100})	0.0005	0.062	7,587	1.87	50.71

For modeling the Q_{100}, two **hydraulic conditions** were characterized:

1) **normal** flow conditions (no backwater, old bridge replaced with planned bike bridge and causeway removed), and

2) **backwater** conditions resulting in a concave upward water surface profile extending upstream from the causeway (representing a greater backwater effect and assuming the log bridge has been replaced by the planned bike bridge).

For normal flow conditions, **slopes** used in the modeling were the same as those determined from the DSC topographic map (S=0.0010 upstream from the county bridge and S=0.0017 downstream from the county bridge). For backwater conditions which would result from the elevated roadway at the lower bridge, a concave upward water surface profile (WSP) would be expected (slope decreases in a downstream direction as the obstruction causing the backwater is approached). This was characterized by reducing the slope in the downstream

half of the study reach to 0.0005. While this value was selected somewhat arbitrarily, it was chosen because it was the midpoint of the range of possibilities (greater than 0 but less than the slope upstream of 0.0010). WSELs at the two upstream cross sections during backwater conditions were determined by projecting upstream from DR15 to the county bridge at S=0.0005 and upstream from there at S=0.0010 to DR05.

The data from the December, 1995, storm is denoted in Table 1 and elsewhere in this report as the "observed" hydraulic condition. As indicated, the water surface profile derived by surveying high water marks ("observed", Fig. 4) varied considerably from that of the channel bed profile within the study reach. Some of the variability may be due to error in precisely locating HWMs or in not being able to find and survey a greater number so that a trend line could be fitted through them on the longitudinal profile. It may also be due to backwater effects which may have occurred during this storm.

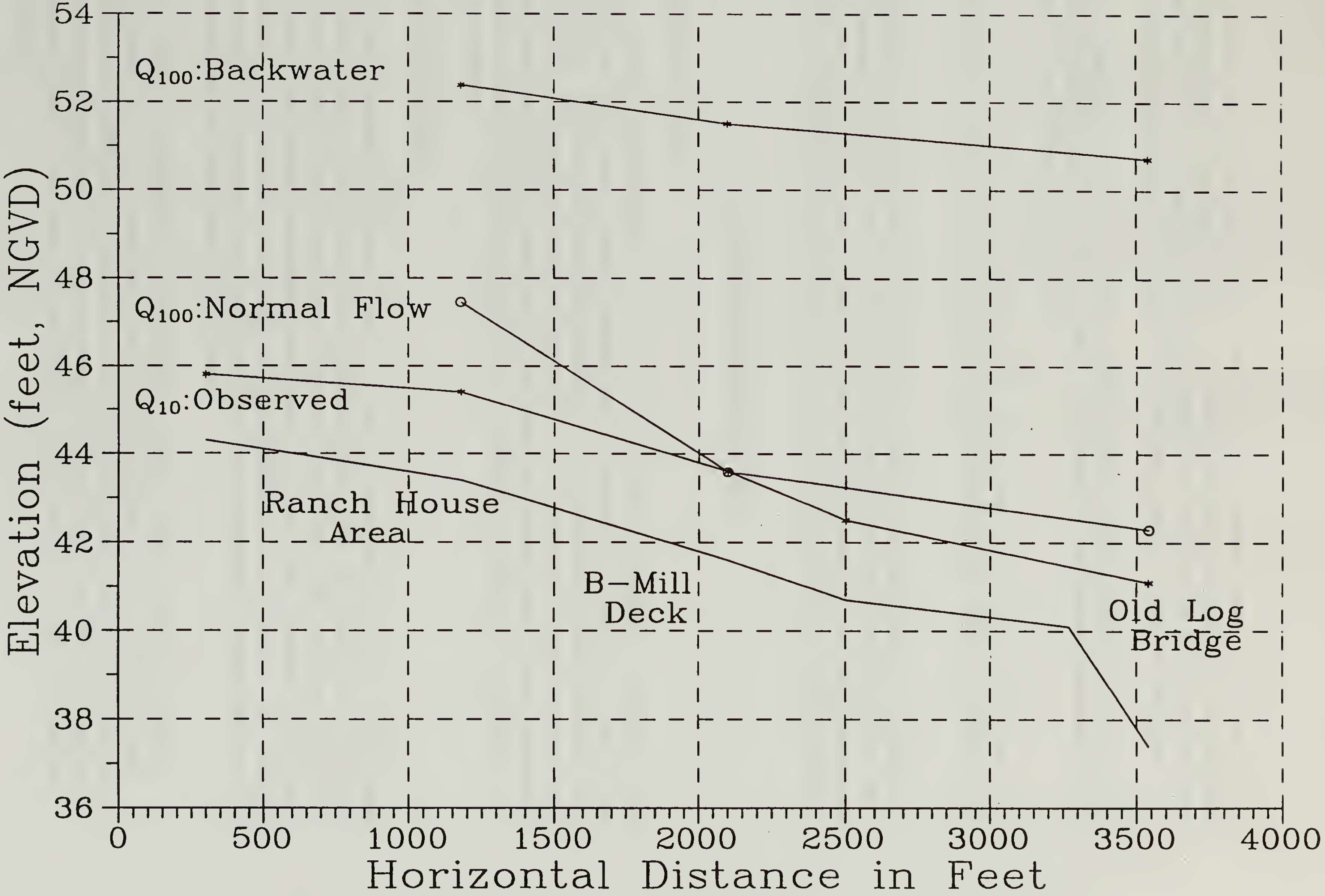
Manning **roughness ("n")** values used in model iterations run to solve for water surface elevation for normal flow conditions at all three cross sections and backwater conditions at cross section DR15 (the downstream control) were determined from tabular values given in Van Haveren (1986) and photographs in Barnes (1967). In these cases, different "n" values were specified for the overbank and channel components of each cross section. For overbank areas, an "n" of 0.50 was used based on normal conditions on floodplains with scattered brush and heavy weeds (Van Haveren, 1986, p. 107). For the Prairie Creek channel, photographs in Barnes (1967, pages 178 and 186) from two rivers of similar slope and appearance exhibited "n" values of 0.059 and 0.060. Accordingly, a value of 0.060 was used for roughness within the Prairie Creek channel.

The weighted averaging done by Flowmaster resulted in all the roughness values of 0.062 which appear in Table 1. All other values of "n" shown in Table 1 are back-calculated values from cases where WSELs were either surveyed in the field (Q_{10} HWMs) or dictated by projecting slopes upstream from DR15 (Q_{100} WSELs at DR05 and DR10 under backwater conditions). Some of the back-calculated roughness values in Table 1 are quite high for this setting, and are likely due to the backwater conditions not well represented by the model used.

Results

Roughness: Initially, the program was run to solve for roughness ("Manning's n"), averaged over the entire cross section, by inputting cross section data along with channel slope (estimated from a longitudinal profile of HWMs from the December, 1995, flood), the observed high water elevations on each cross section from the December, 1995, flood, and the estimated peak flow (5,860 cfs). This yielded unrealistically high values for roughness for DR05 and DR10 (see Table 1, "observed" hydraulic condition, cases 1 & 4), suggesting either: 1) an error in one or more of the input variables (slight errors in one or more of the other components of the Manning formula may create large errors when solving for "n") (Gary Smillie, NPS-WRD, Fort Collins, CO, pers. comm.), or 2) the existence of backwater conditions. As mentioned earlier, the elevated roadway at the downstream end of the study site is a likely cause of backwater during floods. Other cases shown in Table 1 where "n" was back-calculated produced similarly high values (cases 3, 6, & 7).

Figure 4. Longitudinal profile of Prairie Creek showing the ground surface at the top at the left bank and water surface profiles at the Q_{10} (5,860 cfs) and the Q_{100} (14,164 cfs).



At DR15, which is at the very upstream face of the old bridge, the calculated roughness for the Q_{10} (case 7) was unrealistically low (0.029). This was most likely an artifact of constriction effects created at the bridge and the resulting change from sub-critical to critical flow as floodwaters accelerated to pass under the portion of the bridge which spans the floodplain.

Water Surface Elevations: Cross section plots showing water surface elevations (WSELs) for the Q_{10}, the Q_{100} under both normal flow conditions and under the backwater condition for the three cross sections are shown in Figures 5-7.

As shown in Figure 4, the Q_{100}, normal flow WSEL dips in the vicinity of the B-Mill deck (at cross section DR10). This is due to a greater floodplain width and steeper slope at this location.

By comparing normal and backwater WSELs shown in Table 1, the magnitude of the effects of the elevated roadway at the lower end of the study reach can be seen. WSELs at the Q_{100} would be elevated by about 8.4 feet at DR15, 7.9 feet at DR10, and 4.9 feet at DR05 due to the presence of the elevated roadway (assuming the old bridge is replaced with a bike bridge).

FLOODPLAIN DELINEATION

Horizontal Extent of Flooding

Using the Q_{100} water surface elevations (WSELs) computed for the three cross sections for the three high flow scenarios, the regulatory floodplain was outlined on a topographic map of the Davison Ranch (Fig. 3). Under normal flow conditions, the 100-year floodplain would be only slightly larger than that observed during a 10-year flood in December, 1995. As mentioned earlier, and as shown on Figure 3, the effect of the elevated roadway would cause expansion of the floodplain to include a significant portion of the B-Mill deck as well as U.S. Highway 101 and lowlands to the east.

Water Depths and Velocities

Excluding the main channel of Prairie Creek, Q_{100} depths over most of the inundated area range up to about 6 feet under normal flow to over 14 feet under full backwater conditions, which would place about 6 feet of water over the lower part of the B-Mill deck and U.S. 101. The deepest areas are located immediately upstream of the elevated roadway (DR15) and along U.S. Highway 101, where a small channel flows adjacent to the base of the highway fillslope. Generally, the shallowest areas would be located over elevated fill areas and over the natural levee along the left (east) bank of the creek.

As indicated in Figure 8, the presence of the elevated roadway would cause floodwaters to overtop US Highway 101 in the downstream half of the study reach and inundate the lower half of the B-Mill deck. Additionally, the ranch house area would be inundated to depths 5 feet deeper than if the causeway at DR15 were removed. It is likely that effects from the elevated roadway would also result in elevation of Q_{100} WSELs some distance upstream from the study site (private lands)

Figure 5. Davison Ranch cross section DR05 on Prairie Creek showing water surface elevations at the Q_{10} (5,860 cfs) and Q_{100} (14,164 cfs).

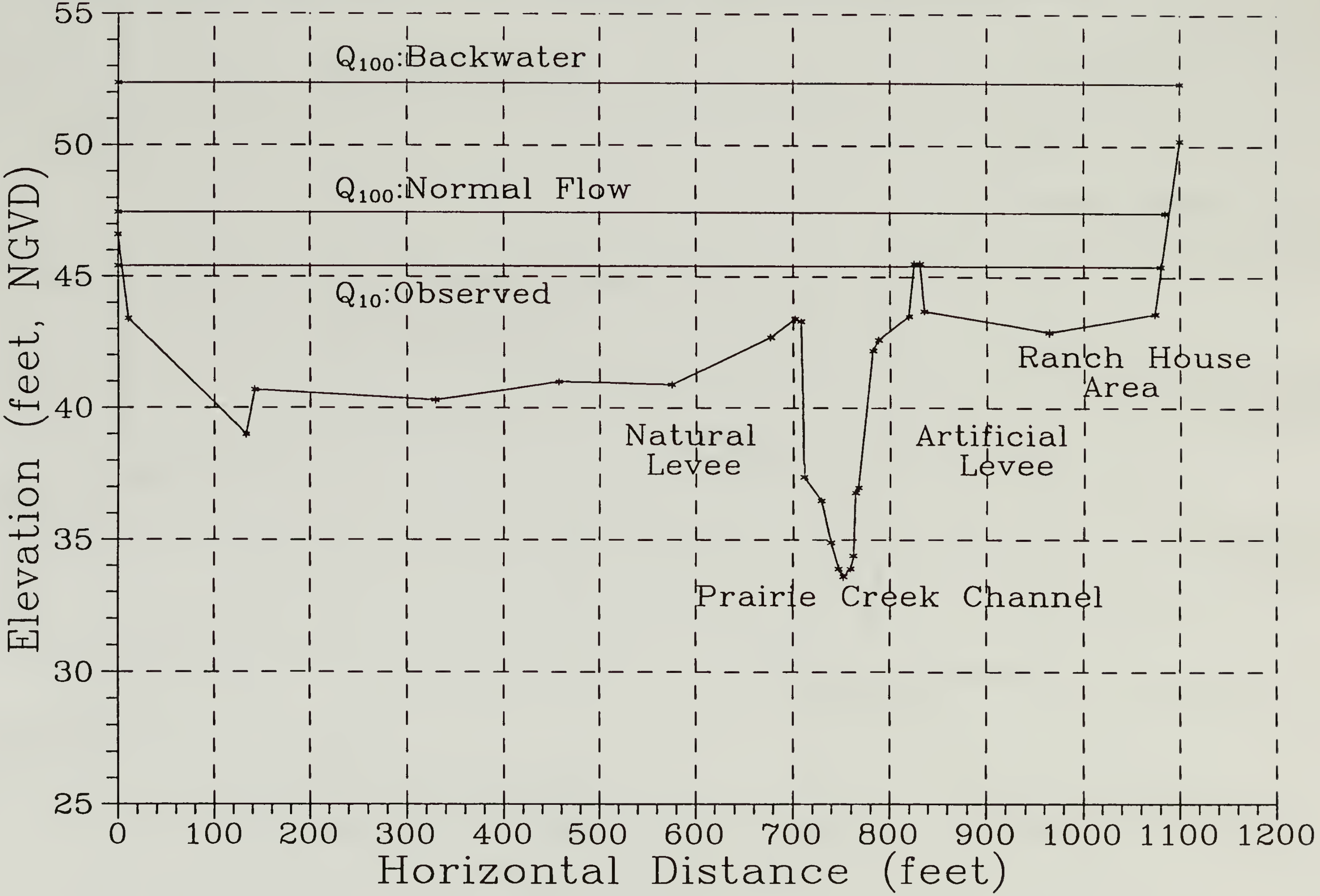


Figure 6. Davison Ranch cross section DR10 on Prairie Creek showing water surface elevations at the Q_{10}(5,860 cfs) and Q_{100}(14,164 cfs).

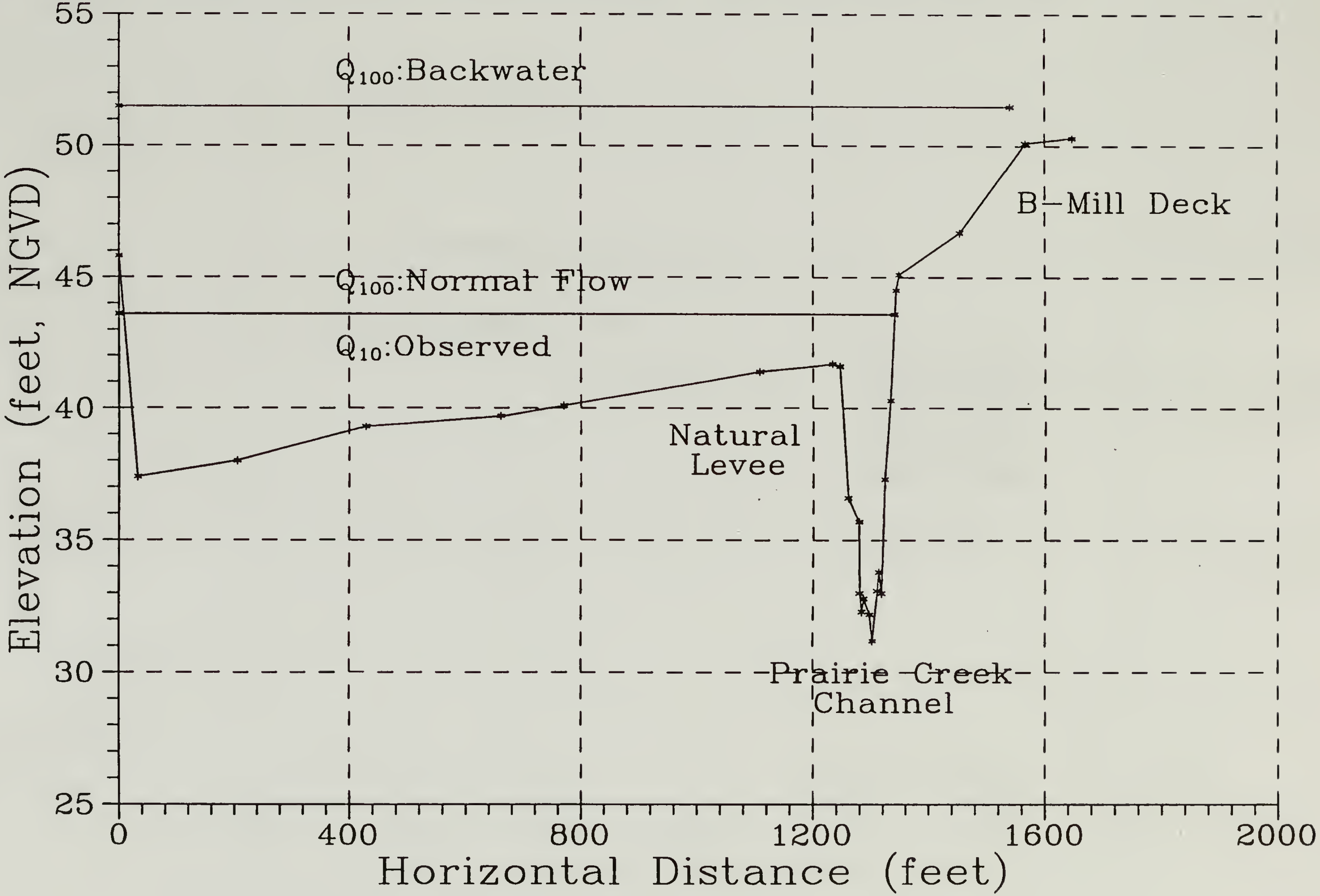
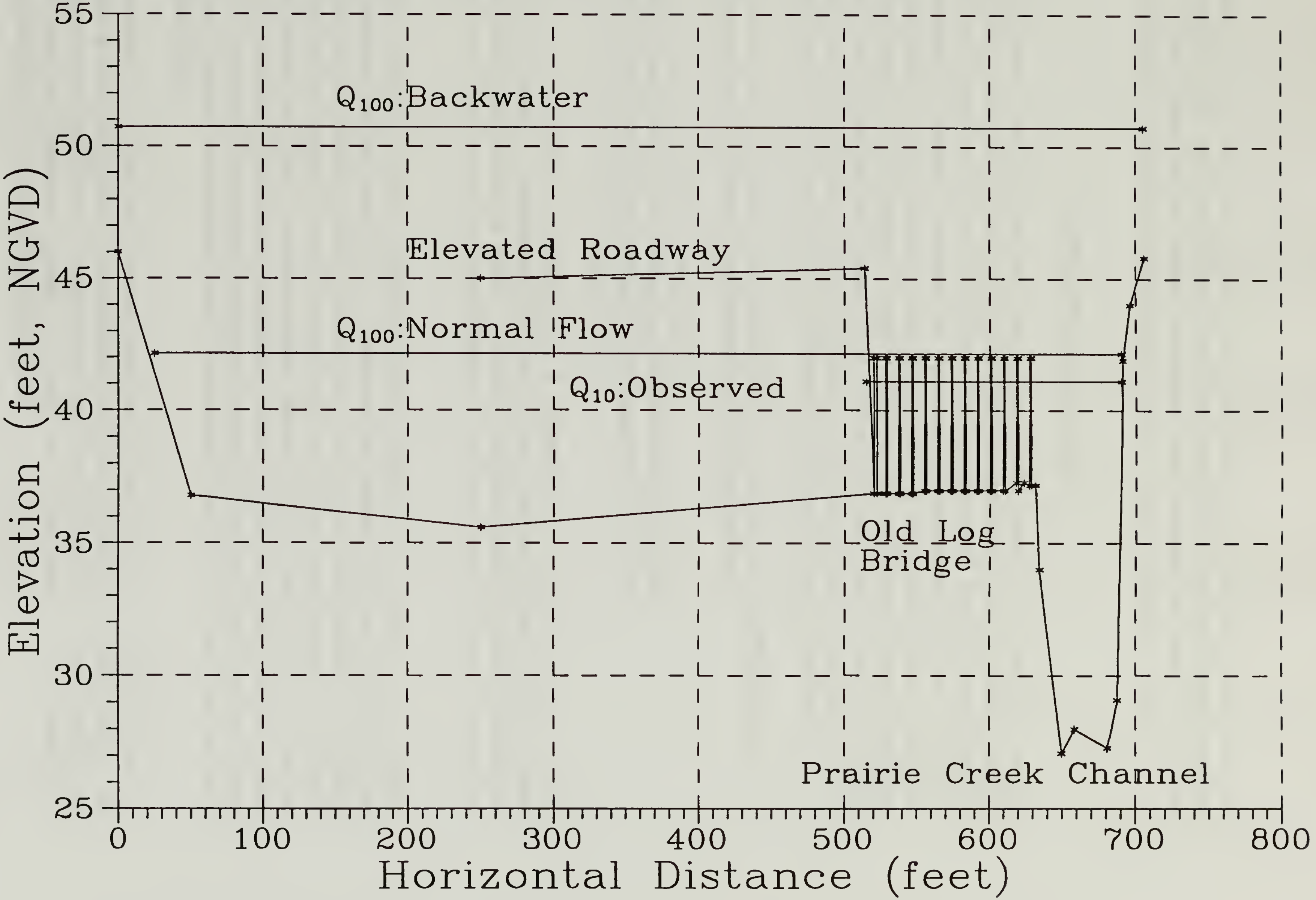


Figure 7. Davison Ranch cross section DR15 on Prairie Creek showing water surface elevations at the Q_{10}(5,860 cfs) and Q_{100}(14,164 cfs).



Model outputs of mean velocities on the three cross sections ranged from over 5 feet per second (fps) (DR15, Q_{10}, "as is" condition) to less than 1 fps (DR10, Q_{100}, backwater condition) (see Table 1). On a cross section, highest velocities would be expected within the channel, with lower velocities on overbank areas (especially through heavily vegetated sections).

SUPPLEMENTAL ANALYSES

Following completion of the foregoing analyses, several more scenarios of flood water surface elevations (WSELs) were developed to: 1) determine the discharge rate at which floodwaters just overtop the causeway at the downstream end of the study area, and 2) characterize the effects of modifying the elevated causeway on water surface elevations during the 10- and 25-year floods. Modifications of the causeway included lowering of the road surface by excavation of fill to allow passage of floodwaters, thereby lowering the peak water surface elevations during floods. These supplemental analyses were done only for assisting in bike bridge design, thus no revision of the floodplain limits (Fig.3) was undertaken based on the results in this section. The following were the scenarios modeled for assisting with bike bridge design:

1) WSELs with the old timber bridge replaced with the bike bridge, as presently designed, and the elevated causeway in its present configuration, at 4 discharge rates:

a) the 10-year flood (5,860 cfs, see Fig. 4)
b) the discharge just below overtopping the causeway (6,425 cfs),
c) the discharge which just overtops the causeway (6,450 cfs), and
d) the 25-year flood (9,000 cfs, see Fig. 4).

2) WSELs for the 10- and 25-year floods for 2 different water surface slopes (minimum probable=0.0010, maximum probable=0.0017) and a "rolling dip" configuration which is about 500 feet long with gentle grades of about 120 feet in length on either side and a level base about 250 feet long in the middle with the elevations of the base at 41, 40, and 39 feet NGVD (Figs. 5-16).

Table 2 shows inputs and outputs for the 16 scenarios modelled. As indicated, flow begins to overtop the causeway at just over 6,425 cfs. With only an additional 25 cfs (to 6,450 cfs), the model predicts that the WSEL jumps by over 2 feet. This is most likely caused by an abrupt change to weir flow, a hydraulic condition which may not be well represented by the model used. At the 25-year flood (Fig. 4), water inundates Highway 101 to a depth of up to 1 foot and flows at a depth of about 2 feet over the causeway. Note that these first three scenarios use a slope of 0.0017, which is the steeper (maximum probable) of the two used in the rest of the scenarios. Use of gentler slopes would result in somewhat higher WSELs than those indicated in Table 2 and Figure 4.

With the causeway dip at the intermediate elevation (40 feet NGVD), a favorable effect on WSELs begins to become apparent. At the gentler of the two slopes, the 10-year flood is well below the highway and would just inundate the bottom chord of the bike bridge. However, the 25-year flood would still inundate Highway 101 under the gentler slope scenario, and would still inundate the bridge at the steeper slope scenario.

With the causeway dip lowered to 39 feet NGVD, all scenarios except the 25-year, gentle slope indicate that WSELs would be below the bottom chord of the bridge. This alternative would go a long way toward easing the flood pressures on the bike bridge at high flows as well as significantly reducing the causeway's effects on upstream flooding.

Table 2. Inputs and outputs for various supplemental hydraulic scenarios for the Davison Ranch bike bridge over Prairie Creek, Redwood National and State Parks.

Scenario Description	Discharge (cfs)	Slope (feet/feet)	Velocity (feet/sec)	WSEL (NGVD)
Bike bridge built, causeway as is, Q_{10}	5,860	0.0017	5.04	41.10
Bike bridge built, causeway as is	6,425	0.0017	5.04	41.52
Bike bridge built, causeway as is	6,450	0.0017	2.84	43.71
Bike bridge built, causeway as is, Q_{25}	9,000	0.0017	3.00	44.39
Rolling dip at 41 feet NGVD, Q_{10}	5,860	0.0010	2.62	42.96
Rolling dip at 41 feet NGVD, Q_{10}	5,860	0.0017	3.16	42.37
Rolling dip at 41 feet NGVD, Q_{25}	9,000	0.0010	2.56	44.39
Rolling dip at 41 feet NGVD, Q_{25}	9,000	0.0017	3.59	43.34
Rolling dip at 40 feet NGVD, Q_{10}	5,860	0.0010	2.66	42.32
Rolling dip at 40 feet NGVD, Q_{10}	5,860	0.0017	3.25	41.66
Rolling dip at 40 feet NGVD, Q_{25}	9,000	0.0010	2.56	44.39
Rolling dip at 40 feet NGVD, Q_{25}	9,000	0.0017	3.65	42.74
Rolling dip at 39 feet NGVD, Q_{10}	5,860	0.0010	2.72	41.64
Rolling dip at 39 feet NGVD, Q_{10}	5,860	0.0017	3.30	44.39
Rolling dip at 39 feet NGVD, Q_{25}	9,000	0.0010	3.09	42.96
Rolling dip at 39 feet NGVD, Q_{25}	9,000	0.0017	3.71	42.09

LITERATURE CITED

Barnes, H.H. 1967. Roughness characteristics of natural channels. U.S. Dept. of the Interior, Geological Survey. Water Supply Paper No. 1849. 213 p.

National Park Service (NPS). 1993. Floodplain management guidelines. US Dept. of the Interior. 14 p.

Redwood National and State Parks (RNSP). 1996. Final development concept plan environmental assessment: Davison Ranch, Humboldt County, California. US Dept. of the Interior. National Park Service. 113 p.

Van Haveren, B.P. 1986. Water resource measurements: a handbook for hydrologists and engineers. Amer. Water Works Assoc. Denver, CO. 132 p